IN PLANE SEISMIC RESPONSE OF IRREGULAR URM WALLS THROUGH EQUIVALENT FRAME AND FINITE ELEMENT MODELS

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ABSTRACT

The assessment of the seismic vulnerability of existing masonry buildings is today a relevant issue for all the earthquake prone countries, and a proper evaluation of their seismic behavior is therefore needed. In this context, the application of verification procedures based on nonlinear analyses is now widespread and requires reliable and computationally efficient modelling strategies. Among the models proposed in literature, the so-called “Equivalent Frame Method” (EFM) has become one of the most successful due to its compromise between accuracy and computational efficiency. According to this simplified approach, each masonry wall is idealized in rigid nodes and deformable portions with a nonlinear behavior (piers and spandrels).

Despite of the large use of these models, to date no validated rules for the EF idealization are available in literature, even for walls with a regular opening pattern; this problem is even more significant when considering existing buildings, where the opening layouts are often irregular. Since no standardized rules are provided, professional engineers can use different criteria for the identification of piers and spandrels, obtaining different outcomes for the seismic assessment.

Within this context, the research here illustrated represents the prelude to the proposal of targeted rules for the EF idealization applicable to different opening layouts. Starting from an initial regular wall configuration, different types of irregularities in the opening pattern are introduced and a comparison between EF models and more accurate Finite Element models is carried on, considering the results provided by these latter as the reference solution.

Keywords: Masonry; Irregular opening pattern; Equivalent Frame method; Finite Element Models

1. INTRODUCTION

Within the context of the seismic analysis of the existing masonry buildings, it is fundamental to have good representative structural models, in order to rightly assess their seismic risk as well as design proper retrofit interventions.

Among the models developed in literature to this aim, the so-called “Equivalent Frame Method” (EFM) represents nowadays one of the most used. It describes the global in-plane behavior of the building and is based on the assumption that the nonlinear response of each wall is concentrated in specific masonry panels defined “a priori” (piers and spandrels) and connected by nodes, usually assumed as rigid.

Thanks to its simplicity of implementation (it can be used both with computer programs specifically oriented to the analysis of masonry buildings and with general purpose software packages) and to its computational efficiency (even for nonlinear analyses), in the last decades the EFM has met great success not only at research level but also for practice engineering aims.

In particular, it is also explicitly recommended by several national and international codes (e.g. Italian

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Building Code (IBC) (M.I.I.TT. 2008) and EC8 (CEN 2005)). Nevertheless, the indications provided by the codes are lacking and insufficient, even regarding the criteria to use for the EF idealization of the walls, that is the first step to afford when using the EFM. This leads to a quite arbitrary application of the method on behalf of the professionals and the analysts who commonly work with it. For this reason, the problem of the reliability and the correct use of these models represents nowadays a topic of great concern that is discussed in literature by several authors (Cattari et al. 2018, Marques and Lourenço 2011).

This paper, in particular, is focused on the study of masonry walls with an irregular opening layout, which are very common in the existing buildings. Opposed to their regular counterparts (walls with openings of the same size and perfectly aligned), the irregular walls present openings misaligned in the horizontal and/or vertical direction or a variable number of openings per story. The earthquake damage observation shows that cracks and failure modes are usually concentrated in specific masonry portions located between the openings aligned in the horizontal and vertical direction: for this reason, the EF idealization of masonry walls is strongly related to the opening layout. It is therefore evident that an irregular opening layout makes difficult and arbitrary the “a priori” identification of the geometries of piers and spandrels.

The current practice in this field is to consider empirical rules based on the observation of damage after past earthquakes (Augenti 2006) and/or calibrated on a few number of experimental tests (Moon et al. 2006) or limited numerical simulations (Dolce 1991) as well as criteria already implemented in computer programs specific for masonry buildings (Lagomarsino et al. 2013). However, rigorous criteria for the identification of piers and spandrels in different wall layouts are still missing. Furthermore, some recent studies on irregular masonry walls devoted to different aims have been developed in the last years and are now available in literature.

The work by Parisi and Augenti (2013) is one of the first to address the issue, proposing a classification of the different types of irregularities recurrent in masonry buildings and some irregularity indexes aimed to quantify them. Other more recent works (Berti et al. 2017, Siano et al. 2017a, Siano et al. 2017b) are aimed to evaluate the accuracy of the EF approach when applied to irregular masonry walls by means of comparisons with more refined modelling techniques (Finite Element models - FE) in linear and nonlinear field. In Calderoni et al. (2017) some suggestions for the application of the EFM to specific cases of irregular masonry walls are provided, even if they are not exhaustive.

Within this general framework, the paper presents some preliminary results which are part of a wider research aimed to systematically validate (or contradict) the criteria available for the EF idealization of masonry walls and to propose targeted rules applicable to different opening layouts. In particular, a systematic comparison between the different rules available is herein illustrated, showing that, especially when dealing with irregular walls, the choice of the geometry for the structural elements represents a key modelling parameter in the EF models.

2. CRITERIA FOR THE EQUIVALENT FRAME IDEALIZATION: STATE OF THE ART

The EF approach is based on the discretization of each masonry wall in nodes (usually assumed as rigid) connecting deformable portions (piers and spandrels), where the nonlinear response is concentrated (Figure 1a).

In the literature several criteria are proposed to define the geometry of such structural elements, all of them related to the opening layout characterizing the wall.

More in detail, the geometrical definition of spandrels in presence of regular walls can be easily obtained by considering the portions of masonry included between two vertically aligned openings. However, in case of not perfectly aligned openings no specific rules supported by numerical or experimental research can be found. An empirical rule proposed in Lagomarsino et al. 2013 is to conventionally assume a mean value for the height of spandrel elements as a function of the overlapping part between the openings at the two levels; when no overlap is present or the opening lacks at all, it is suggested to assume the portion of masonry as a rigid area.
Figure 1. (a) EF idealization of a masonry wall - pier’s effective height according to Lagomarsino et al. (2013); different criteria for the pier’s effective height: (b) Dolce (1991), (c) Augenti (2006), (d) Moon (2006)

Considering masonry piers, the principal problem is the definition of their effective height; in particular, uncertainties arise in presence of openings with different heights at the same floor (horizontal irregularity) or in the case of external piers, where the formation of inclined cracks starting from the opening’s corners, documented by the observed damage, has to be taken into account. Regarding this aspect, there are several criteria proposed in literature and based on different principles. Dolce (1991), on the basis of a series of Finite Element (FE) analyses on 20 different pier–spandrel systems, proposed a simplified formula, derived by considering a principle of statistic equivalence between the elastic stiffness of EF and FE models. Moreover, a limit inclination of 30° was introduced for the cracks that start at the right or left corner of the openings and propagate toward the opposite pier edges. The proposed formula is a function of a geometrical parameter (h’ in Figure 1b) defined as the distance between the midpoints of the lines connecting the vertices of two consecutive openings; according to what observed about the cone diffusion of the cracks, these lines have a limit inclination of 30°. The final effective height of the pier (H_{eff}) is then obtained by properly modifying h’, accounting for the global geometry of the considered level of the wall.

In the TREMURI software (Lagomarsino et al. 2013) a criterion based on the Dolce’s proposal but without the limitation on the maximum inclination of cracks has been implemented. While the two aforementioned criteria do not depend on the direction of the seismic action, other rules, taking into account that the cyclic nature of the earthquake motion can induce a different failure pattern depending on the orientation of the seismic forces, lead to the definition of two different capacity models for the same wall. In particular, in Augenti (2006), by observing the damage occurred in masonry buildings after past earthquakes, it is proposed to assume the pier’s effective height equal to the height of the opening which follows the pier in the direction of the seismic load (Figure 1c).

In addition, considering the results of a quasi-static lateral loading tests on a full-scale 2-story URM building (Yi et al. 2006), Moon et al (2006) proposed a pier’s effective height equal to the height over which a compression strut is likely to develop, at the steepest possible angle (Figure 1d).

It is stressed that the criteria on wall's discretization discussed above have been mainly developed for masonry walls with quite regular opening patterns, and not enough comparative studies justify their application to existing URM buildings with very irregular opening layouts. Within this heterogeneous framework it emerges that, since for masonry spandrels not exhaustive indications are present and for the pier’s effective height many different criteria are available, the EF idealization of a given masonry wall is quite arbitrary. As a consequence, given the same architectural configuration it is possible to obtain, depending on the chosen criterion, different structural models. Although these differences are more evident when considering irregular walls, even in a regular wall some uncertainties may arise, mainly regarding the effective height of the external piers (Figure 2).
It is evident that when different structural models are considered for the same wall, different global responses can be obtained, thus implying critical issues on the outcomes of the seismic verifications, as well illustrated in the following sections.

3. ADOPTED METHODOLOGY

The defined methodological approach is based on the execution of numerical nonlinear analyses and considers the results of a Finite Element (FE) model in which masonry is modelled as a continuum equivalent material as the reference solution. In particular, it can be summarized as follows:

- definition of the configurations to analyse (i.e. walls with different irregular opening layouts);
- then, for each defined wall configuration:
  - individuation of the possible EF idealizations on the basis of the criteria available in literature;
  - definition of a FE model, whose results are considered as the reference solution;
  - execution of nonlinear static analyses (NLSA) on the defined numerical models;
  - ex-post identification of a further EF model on the basis of the results of the FE analysis;
  - comparisons of the results in terms of global response (i.e. in terms of pushover curves: total base shear $V_b$ vs. mean top displacement $d_{top}$), local response and damage pattern.

As the local response concerns, the aim is to compare generalized forces (normal force, shear force, bending moment) and displacements associated to specific sections of the structure. While in the EF model these results are straightforward, in the FE model it is necessary to define fixed sections (located on vertical and horizontal alignments identified in the wall – see Figure 3) where the integration of the nodal stresses and the choice of representative displacements have to be done.
3.1 Definition of the wall configurations

As shown in Figure 4, all the irregular configurations have been defined starting from a *regular* wall, characterized by openings of the same size and perfectly aligned. This is motivated by the necessity to validate the methodology on a simple case in which less uncertainties are expected and where the interpretation of the results should be easier. The geometry of the considered regular wall refers to the so-called “Door wall”, which is part of a two-storey brick masonry building prototype experimented with a quasi-static cyclic test (Calvi and Magenes 1994). Although the rigorous numerical simulation of such test is not the aim of the research, it has been selected in order to have a reference solution to check the reliability of the results obtained with the adopted numerical models when mechanical parameters compatible with the masonry characterizing the prototype are adopted.

![Figure 4. Defined case studies](image)

Regarding the irregular configurations, different types of irregularities, according to the classification proposed in Parisi and Augenti (2013) have been considered, each one denoted with a specific letter:

- *offset irregularity*, indicated with letter A in case of offset irregularity in the horizontal direction and with letter C in case of offset irregularity in the vertical direction;
- *horizontal irregularity*, indicated with letter B;
- *vertical irregularity*, indicated with letter D;
- *different number of openings per storey*, indicated with letter E.

The name of each configuration (indicated in bold in Figure 4) reflects the types of irregularity that characterize it. Moreover, the geometry of the irregular walls has been defined in order to explore the situations in which, as observed in §2, several uncertainties or even lacks are present in literature. While Figure 4 illustrates the more general research plan designed, in §5 only a subset of results is presented, mainly related to BD and A type irregularities.

3.2 Numerical models

The nonlinear static analyses on the EF models have been carried out using the research version of the Tremuri program, which is a software specifically oriented towards the seismic assessment of masonry buildings (Lagomarsino et al. (2013)). In this case masonry panels are modelled as nonlinear beams with lumped plasticity and a piecewise-linear behaviour (Cattari and Lagomarsino 2013), which allows to describe the nonlinear response until very severe damage levels (from 1 to 5) through progressive strength decay in correspondence of assigned values of drift.

The FE models have been realized by means of ABAQUS v. 6.14 and by using 8-node isoparametric solid elements. Moreover, masonry has been modelled as a continuum equivalent material through the *concrete damaged plasticity* model (Lubliner et al. 1989), which is an isotropic phenomenological model that uses concepts of damaged elasticity in combination with tensile and compressive plasticity. The total damage for cracking in tension and crushing in compression is monitored through two specific damage variables ($\alpha_T$ and $\alpha_C$ respectively), which can vary from 0 (undamaged material) to 1 (completely damaged material).
Table 1. Mechanical parameters of the masonry used in the numerical models.

<table>
<thead>
<tr>
<th></th>
<th>$\rho$ [kg/m$^3$]</th>
<th>E [MPa]</th>
<th>G [MPa]</th>
<th>$c_{eq}$ [MPa]</th>
<th>$\mu_{eq}$ [-]</th>
<th>$f_{bt}$ [MPa]</th>
<th>$f_c$ [MPa]</th>
<th>$f_t$ [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>EFM</td>
<td>1750</td>
<td>1800</td>
<td>600</td>
<td>0.178$^{(*)}$</td>
<td>0.45$^{(*)}$</td>
<td>1.22</td>
<td>6.2</td>
<td>-</td>
</tr>
<tr>
<td>FEM</td>
<td>1750</td>
<td>1800</td>
<td>750</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>6.2</td>
<td>0.22</td>
</tr>
</tbody>
</table>

Notes: $\rho$ density; E elastic modulus; G shear modulus; $c_{eq}$ equivalent cohesion; $\mu_{eq}$ equivalent friction coefficient; $f_{bt}$ blocks tensile strength, $f_c$ compressive strength of masonry; $f_t$ tensile strength of masonry.

$^{(*)}$ the values of $c_{eq}$ and $\mu_{eq}$ are obtained according to Mann and Muller (1980), starting from the correspondent local parameters of the mortar joint and modified as follows (in order to account for the actual geometry of the masonry pattern, described by the parameter $\phi$, here assumed equal to 0.5):

$$c_{eq} = \frac{c}{1 + \phi \mu}; \quad \mu_{eq} = \frac{\mu}{1 + \phi \mu}$$

Even though the final aim of this research is to consider different types of masonry (regular and irregular), in these first analyses the mechanical parameters used (see Table 1) refer to brick masonry and are consistent with those of the prototype building tested in Pavia (Anthoine et al. (1995)).

In the EF models, according to the masonry type considered and to the failure modes which can occur, it is possible to use specific failure criteria for the masonry panels. In particular, herein, for the flexural failure the criterion provided in IBC (M.I.I.TT. 2008) has been considered, while for the shear failure the criterion proposed by Mann and Muller (1980), which is more appropriate for regular brick masonry (where stair-stepped shear failures can be activated), has been chosen. However, it would also be acceptable to use the criterion proposed by Turnsek and Cacovic (1971), even if it is more specific for irregular masonry (almost isotropic); in fact, its parameters can be calibrated in order to reproduce the predictions of the Mann and Muller’s criterion within the considered range of variation for the normal stress.

Regarding the FE model, the regular wall has been already simulated in previous works by using anisotropic constitutive laws specific for brick masonry (Gambarotta and Lagomarsino 1997, Cattari 2007, Calderini et al 2009) and obtaining a good agreement with the experimental results. In this research, the aim is then to explore the possibilities of the aforementioned isotropic phenomenological model, which will be used in a more extended way for all the configurations, simulating also, through a proper calibration of its parameters, the different masonry types which is intention to consider.

Furthermore, since the EF and the FE models work at different scales and are based on different mechanical parameters, a preliminary calibration of the failure surface used in the FE model is needed in order to reproduce the failure domain of the structural elements used in the EF one.

To this aim, some preliminary analyses on masonry panels with different slenderness ratios $\lambda$ were performed. The results of the calibration, which was realized considering the range of variation for the normal stress of our interest, led to a final value for the tensile strength of masonry equal to 0.22 MPa, as reported in Table 1.

With regard to the structural details, while in the case of the regular configuration two rigid steel beams located at the diaphragm level have been modelled (to simulate the effects induced in the experimental test by the steel profiles used for the application of the horizontal displacements), in the case of the irregular configurations, RC ring beams have been considered. In this latter case, a preliminary calibration on the regular wall with the results of the FE model has been realized, in order to choose the effective length to use for the RC ring beams in the EF models.

4 REGULAR CONFIGURATION: RESULTS

Once calibrated the numerical models, the NLSA on the regular wall have been carried on; the obtained pushover curves are shown in Figure 5a, together with the results of the FE simulation performed in Calderini et al (2009) by using the constitutive model proposed in Calderini and Lagomarsino (2008). First of all, considering the results of the different EF models, it comes out that the geometry assumed for the structural elements actually influences the global response in terms of both stiffness and strength, even in case of this regular wall.
In general, a good correspondence is observed between the predictions of the considered numerical models; these last are in good agreement also with the results of the experimental test, keeping in mind that it was a cyclic test, while the analyses here performed are monotonic. Furthermore, the isotropic FE model herein used provides results which are consistent with the simulation performed in Calderini et al (2009), also considering that in the latter an anisotropic constitutive model specific for brick masonry and calibrated on the same mechanical parameters (Anthoine et al 1995) was used.

The comparisons in terms of local response show that, even if the good agreement between FE and EF models observed at the global scale is in general confirmed, the effective height chosen for the piers actually affects in the EF models the generalized forces obtained, both quantitatively and qualitatively. The comparisons presented in the following refer to the EF model where piers and spandrels are identified ex-post from the results of the FE analysis (Figure 5b). This case is interesting because here the uncertainty about the “a priori” identification of piers and spandrels, which is intrinsic in the EFM, is removed and the geometry of the structural elements is exactly the same in the two models.

In Figure 6 the evolution of the generalized forces acting at the base sections of the wall is shown. In the initial phase a quite perfect agreement in the redistribution of the stresses among the three piers at the ground floor is observed, while more differences are detected when the response becomes strongly nonlinear, even if the general trend is substantially the same in the two models.

Regarding the comparisons in terms of generalized forces (Figure 7), 4 fixed and increasing values of the top displacement have been considered (see Figure 5b). These comparisons show again a good agreement in the predictions of the two models, considering both the initial response (Step 1 and Step 2) and a more advanced nonlinear phase (Step 3 and Step 4).

By observing the results of the FE model, it emerges that the wall subjected to the horizontal forces actually behaves like a frame, allowing to distinguish the parts of the wall corresponding to the rigid nodes and those representing the pillars and the beams of the correspondent equivalent frame.
Figure 7. Comparison between the FE model and the “ex-post” EF model: shear force on alignment C3 (a), shear force and bending moment on alignment C2 (b1,b2), normal force and shear force on alignment R1 (c1,c2)

More in detail, in the case of the vertical alignments the bending moment diagram is linear in the portions of the wall corresponding to the pillars of the frame (the piers in EF model), thus allowing the identification of the effective height of piers and its comparison with that suggested by the different criteria used. Considering the horizontal alignments, more differences between the 2 models are detected. This is not surprising, since in the EF modelling the behavior of spandrels is characterized by more uncertainties than that of piers; this is because spandrels have lately been recognized to play a significant role on the load bearing capacity of masonry walls, and so they have started to be investigated more in detail only recently. In particular, it is observed that the FE model provides higher values of normal force than the EF one. Indeed, while in the FE model the coupling between masonry and the rigid elements placed at the level of the diaphragms is realized in a single point and all the remaining section is made of masonry, in the EF model the spandrels are modelled as 2-node beams and the rigid element is connected with masonry exactly in the nodes at their end sections. It is therefore evident that in the EF model the rigid elements give a higher contribution to the bearing of the normal force, thus producing a lower stress in masonry.

Figure 8. Comparison between the FE model and the “ex-post” EF model of the regular wall: horizontal displacements for the alignment C1 (a) and vertical displacements for the horizontal alignment R2 (b)
Despite the higher differences in the normal forces, the shear sustained by the spandrels is in quite good agreement, being the difference explained by considering that in the EF model the actual uniform distributed vertical loads are transformed into concentrated forces acting on the nodes at the end sections of each spandrel element.

The comparisons in terms of displacements (some of them are shown in Figure 8) confirm that in the FE model there are some portions of masonry working like rigid nodes and others which are more deformable, thus providing deformed shapes which are similar in the 2 models. This can be clearly seen, especially when considering the response of the model in advanced nonlinear phase.

In case of horizontal alignments, the more significant differences between the two models are consistent with those detected also in terms of generalized forces; however, the EF model seems able to correctly predict the deformed shape both qualitatively and quantitatively.

5. FIRST RESULTS ON THE IRREGULAR CONFIGURATIONS

Once validated the defined methodological approach on the regular configuration, the same procedure has been applied in the case of the irregular ones. It is evident that, differently from the regular case, which was symmetric, here the nonlinear static analyses have to be performed considering both the positive and the negative verse, and the response may be significantly different in the two cases.

In the following only the results of the comparisons in terms of global response and damage pattern are shown, being those about the local response still work in progress.

In Figure 9 the results of the configuration BD in terms of global pushover curves (positive verse) are shown. In this case, in addition to the EF models obtained with the application of the criteria described in §2 (Figure 2), a further model (named “No window”) where the presence of the little opening is neglected has been considered, being this latter one of the possible modelling choice in such situations. Here the pier’s effective height is determined according to Lagomarsino et al. (2013).

First of all, considering the responses provided by the EF models, it is observed that the scatter of the results is much more significant than in the case of the regular configuration.

![Figure 9. Configuration BD: (a) global pushover curves (positive verse) and (b) damage pattern (d_{top}=16.5 mm) for the EF model defined according to Augenti (2006)](image)

More in detail, the model obtained with the application of Dolce’s criterion presents a lower global stiffness when compared to the other EF models: this is due to a smaller extension of the portions idealized as rigid nodes with respect to the other models. Considering the post-peak response, it can be seen that all the EF models predict a drop of strength (d_{top} about 15-18 mm); however in the models associated with the rules proposed in Augenti (2006) and Moon et al (2006) the reduction of the base shear is more significant than in the other models: this is caused by the premature failure of the squat central pier which forms at the ground floor (Figure 9b). In the EF models, indeed, the failure of masonry panels is governed by the reaching of fixed values of drift. As a consequence, it is evident that rather squat panels will fail for very low values of the horizontal displacement, thus potentially affecting the global ductility of the system and consequently the outcomes of the seismic verifications.

In the other EF models, where the piers at the ground floor are not so squat, this does not happen.
Comparing the pushover curves of the EF models with the FE curve, which herein is considered as the reference solution, it can be seen that the “No window” model gives a good prediction of the maximum strength, while the model according to Lagomarsino et al. (2013) provides a better description of the whole global response, even considering the post-peak phase.

With regard to the damage pattern activated in the FE model (Figure 10a), the inclination of the cracks at the ground floor clearly indicates the activation of a unique pier, thus suggesting to neglect the presence of the little opening. Comparing these evidences with the damage pattern resulting in case of configuration B1 (Figure 10b), where the opening at the ground floor has intermediate dimensions between the little opening of configuration BD and the door opening of the regular wall, it is evident in this case the activation of two different piers.

On the basis of these results, future analyses will be addressed to the individuation of the limit dimensions of the opening that represent the transition between the necessity to consider or to neglect the opening in the structural model. However, for the moment it is difficult to understand which one of the considered EF models provides the best structural response when compared to the reference solution: the comparisons in terms of local response will allow to be more exhaustive about the topic.

Considering the configuration A2, the aim is to understand if the spandrel between the openings on the right side of the wall has to be considered or not in the structural model. Two different EF models have been considered: in the first the spandrel is neglected (consistently with the proposal by Lagomarsino et al. (2013)), while in the second it is present. This choice is motivated by the fact that, since no horizontal irregularity is present, the different criteria for the determination of the pier’s effective height are expected to provide quite similar results, as in the case of the regular wall.

It is observed, again, that the chosen modelling hypotheses in the EF model affect the obtained global response (Figure 11). However, since in the analysed walls the spandrels are coupled to RC ring beams, the scatter between the responses predicted by the two EF models is not so pronounced.

Considering the analyses in the positive verse the differences between the two EF models are mainly concerning strength, while in case of negative verse the pushover curves differ both in terms of strength and ultimate displacement capacity. In the first case, in particular, even if in both models the activated global mechanism is the same (soft storey at the ground floor), the damage pattern associated to the two models (Figure 12) presents some differences (in addition to the obvious ones related to
the spandrels): when the spandrel is not modelled, damage is mainly concentrated at the ground floor, while when it is present the damage is more diffused and involves also the piers at the second floor.

Regarding the comparison with the results of the FE analysis, the EF model in which the spandrel is present seems to provide a better response, both in terms of global curve and activated damage pattern. As seen in Figure 12c, indeed, the portion of masonry between the two openings is actually interested by damage, and it seems not correct to model it as a rigid node. It will be interesting to analyse further configurations with a more significant vertical misalignment between the openings at the two levels, in order to understand if there is a limit situation beyond which it is more correct not to model the spandrel between them and to define standardized criteria also for such case of irregularity.

6. CONCLUSIONS

In the paper a review of the main criteria proposed in literature for the EF idealization of masonry walls has been presented, showing a lack of rigorous and validated rules which makes quite arbitrary the “a priori” identification of piers and spandrels and leads to the possibility of having different structural models for the same wall. Within this context, some results of a wider research aimed to propose targeted rules applicable to different opening layouts have been illustrated. The defined procedure has been validated on a regular wall for which also experimental results are available and have been used to assess the reliability of the adopted numerical models. The predictions of the EF and FE models show in general a good agreement. Furthermore, also the introduction of the rigid nodes, which represents one of the most significant simplifications used in the EF approach, finds a confirmation in the behaviour of the FE model in terms of local response. The same procedure has then been applied to the defined irregular configurations, highlighting that the indiscriminate use of the rules available for the EF idealization can lead to critical and unrealistic situations in the structural models, such as the presence of very big rigid nodes and/or very squat piers. Although for the moment the results obtained for the irregular configurations have to be considered as preliminary and not yet conclusive in defining standardized criteria, it has emerged that the choice of the geometry for the structural elements represents a key modelling parameter in the EFM. It can significantly affect the predictions of the structural models in terms of global response, and thus the outcomes of the seismic verifications, especially in case of irregular walls but also when considering walls with a regular opening pattern.

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8. REFERENCES


